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STONE MATRIX ASPHALT
THE WISCONSIN EXPERIENCE

FINAL REPORT

WISCONSIN
TECHNOLOGY
ADVANCEMENT
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16. Abstract

In 1991, the Wisconsin Department of Transportation and the asphalt paving industry in the state decided to construct a trial installation of an asphalt paving technology that had garnered the interest of a group of professionals that had recently toured Europe. The success of that initial trial was the basis of the decision to conduct a thorough evaluation of Stone Matrix Asphalt (SMA). Subsequently, six projects were constructed at various locations around the state. Each project contained six test sections utilizing various fiber and polymer modified SMA mixes. Besides evaluating the various types of SMA's, this research effort also contained elements in it to allow evaluation of the impact of aggregate size and aggregate hardness on the effectiveness of the SMA mixes. These projects were constructed over a period of three years, with the last one constructed in 1994. All of the projects were evaluated from the standpoint of the ease of construction and performance after five years. While the ease of construction was to be evaluated mostly on a subjective basis, the performance measures were established to be objective and measurable. The performance measures were: amount of cracking; friction characteristics; overall pavement distress; amount of rutting; noise impacts; and ride.

At the completion of the five-year evaluation period, SMA's are performing better than the standard asphaltic concrete pavements in some important areas, i.e., crack and distress generation. However, their overall cost-effectiveness compared to a standard dense-graded asphalt pavement was not evaluated, therefore is unknown. The trend suggests that a SMA pavement may have a longer service life, although no hard evidence is forthcoming from this project at this time.

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A. INTRODUCTION

1. History

The ever-increasing traffic volumes, including increased truck traffic and higher tire pressures, are putting greater and greater stresses on our asphalt concrete pavements. These stresses manifest themselves primarily in the form of rutting. In order to handle these increased stress levels and to reduce the attendant rutting, the Wisconsin Department of Transportation (WisDOT) and other state DOT's have been endeavoring to design stiffer, more rut-resistant asphalt concrete pavements. By the late 1980’s and early 1990’s these approaches had generally been considered moderately successful; however, the search for a more durable and more cost/effective asphaltic concrete pavement remained an on-going effort.

Observations made by a group of state and federal highway officials and contractors from the United States on tour to Europe in 1990, brought a unique asphaltic concrete pavement to the attention of state DOT's. This unique asphalt pavement had demonstrated its ability to withstand the heavy truck loadings and resist the wear of the super wide, single truck tires and the studded tires used or once used throughout Europe.

This asphaltic concrete pavement, called Stone Matrix Asphalt (SMA), is characterized by gap-graded aggregates, high asphalt contents and polymer or fiber additives for stabilizers. It produces a densely compacted asphalt pavement-wearing surface that has achieved prominence throughout northern and central Europe. Since the tour to Europe, it also garnered considerable attention in the United States and Canada.

In early August of 1991, based on the reported performance of SMA, WisDOT decided to construct a trial section of this new type of asphaltic concrete (AC) pavement. The trial was conducted on I-94 in Waukesha County near Milwaukee. As a result of the success of that trial, which established the ability to construct SMA using conventional equipment typically available in the United States, WisDOT decided to evaluate the potential for conducting an in-depth SMA research study and develop a work plan to govern that study.

2. Study Development

a) Industry Involvement

From the start, this project was purposefully conducted as a partnering effort between the state and federal highway agencies and the asphalt paving industry. On August 22, 1991, a meeting was held to discuss the merits of developing a research study to investigate the effectiveness of SMA pavements in addressing the distress and rutting impacts imparted by modern traffic on asphalt concrete pavements. Members of the Federal Highway Administration, the WisDOT and the Wisconsin Asphalt Pavement Association attended this meeting. The outcome of this meeting was a commitment of all parties to proceed with the development of a logical plan for a major research study to investigate SMA's in Wisconsin.

b) Establishing Study Objectives

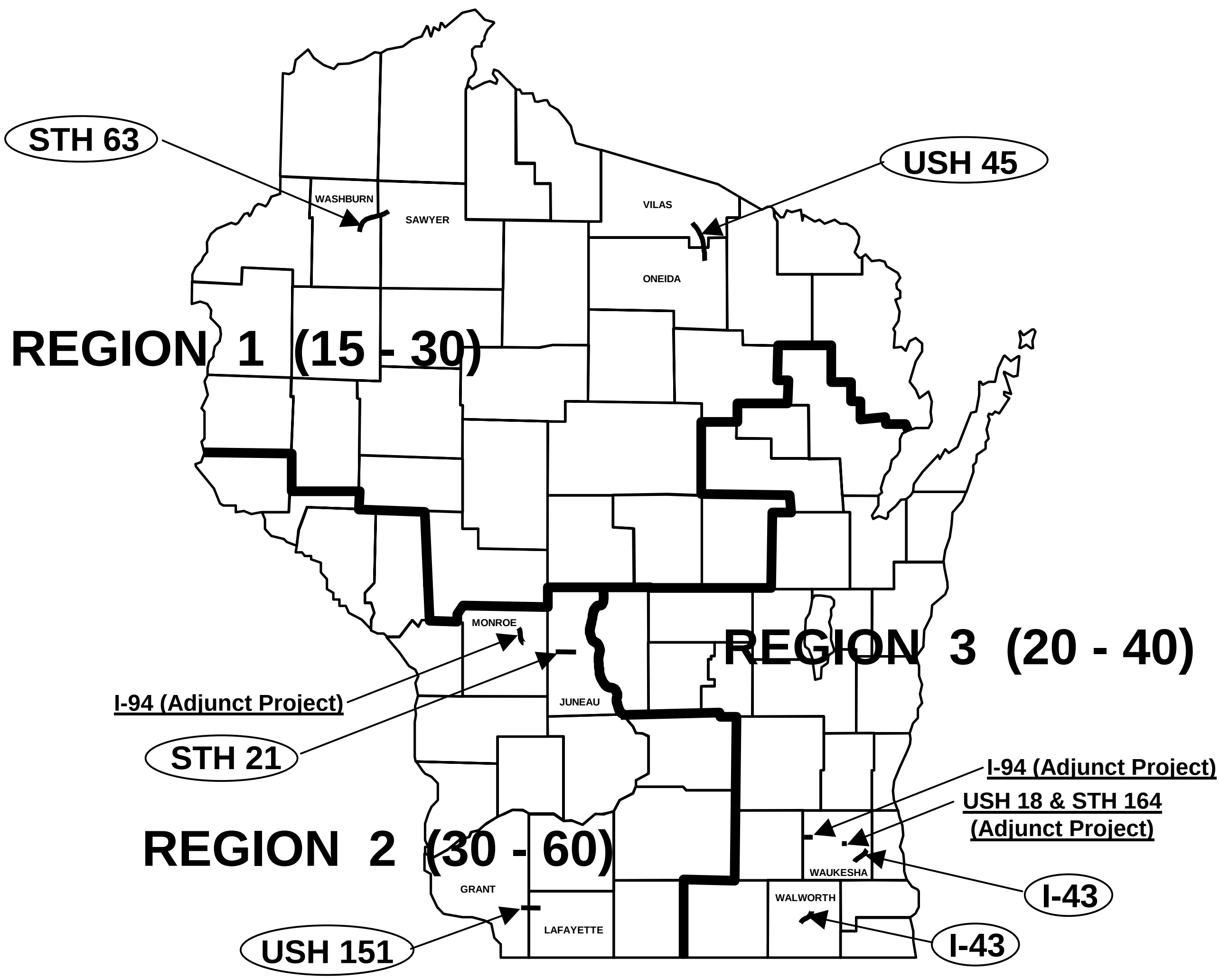
The purpose of the study, as defined at that meeting and in the research work plan, was to evaluate the relative ease of construction of various SMA pavement types and to compare their performance against the standard asphaltic concrete pavement. The SMA pavements to be evaluated would contain a range of different stabilizers, including both cellulose and mineral fiber stabilized mixes and thermoplastic and elastomeric polymer stabilized mixes. Six main parameters were used to evaluate the effectiveness of each SMA pavement type from the standpoint of overall performance: (1) the amount of cracking; (2) friction; (3) overall pavement distress (PDI); (4) the amount of rutting; (5) noise impacts; and (6) ride (using the International Ride Index or IRI).

The guidelines established at that meeting for conducting the study were: (1) Evaluate the role of aggregate hardness on the effectiveness of SMA's; (2) Select two projects on high volume roads in each of the aggregate hardness regions of the state; (3) Construct all test sections on each of these projects a minimum length of 4000 feet (1200 meters), one-lane wide; (4) Evaluate the range of different SMA stabilizers, including fibers and polymers; (5) Construct the projects with 3/8" (9.5 mm) and 5/8" (16 mm) maximum sized aggregates; (6) Construct the projects as soon as possible (during the 1992, 1993 and 1994 construction seasons); and, (7) Monitor the pavement performance of each project for a minimum period of five years following their construction.

c) Selecting Study Projects

Based on the objectives described above, a list of sixteen potential projects was developed for possible inclusion in the research study. Concurrently with the project selection process the state hardness regions, based on Los Angeles (L.A.) Wear values, were defined. This was accomplished by accessing the records of L.A. Wear testing for aggregate sources around the state, and, using these values, dividing the state into three hardness regions. These regions included: Region One, located in the northern half of the state, generally characterized by igneous gravels with L.A. wear values between 15 and 30; Region Two, in the southwestern part of the state, consisting of softer, more absorptive dolomitic crushed stone or gravels with hardness values between 30 and 60; and, Region Three, in the southeastern part of the state, generally consisting of limestone/dolomite stone or gravels with hardness values between 20 and 40. WisDOT currently specifies a maximum L.A. Wear of 45%, which is consistent with the national guidelines. Subsequently, six projects were chosen for the research study, two located in each of the three hardness regions. See Figure 1 on the following page for the location of the hardness regions and the projects located in each.

FIGURE 1: WISCONSIN HARDNESS REGIONS
(Based on L.A. Wear Results)



B. DESCRIPTION OF STUDY PROJECTS

1. Project Locations

As can be seen in Figure 1, the primary projects are scattered around the state, two located in each "Hardness Region", as mentioned previously. In addition to these six primary projects, three other projects were included in the study. These three projects were to be monitored to the extent possible, but were only considered adjunct or supplemental to the six primary projects. These three adjunct projects are also shown in Figure 1. Results from the three supplemental projects will not be specifically reported on as their construction was not closely monitored, nor was their performance followed as closely as that of the primary projects. However, some interesting aspects relative to SMA's were learned from their inclusion in the study.

Two of the three adjunct projects had been constructed in the summer of 1991 and are located in Waukesha County in southeastern Wisconsin. One of these was the first SMA project constructed in the state and was placed in the westbound driving lane of I-94, west of the interchange with STH 67. The other is located in the city of Waukesha at the intersection of Blue Mound Road (USH 18) and STH 164. The third adjunct project, constructed in the summer of 1992, is located in the central part of the state on I-94, just north of Tomah in Monroe County.

Following, then, are the six primary projects organized by their respective hardness regions and the three adjunct projects involved in this research study:

Primary Projects -

Region One:

1) USH 45, Vilas and Oneida Counties, Eagle River to Three Lakes

2) USH 63, Washburn and Sawyer Counties, Springbrook to Hayward

Region Two:

3) USH 151, Grant and Lafayette Counties, Platteville to Belmont

4) STH 21, Juneau County, Monroe County Line to Necedah

Region Three:

5) I-43, Waukesha County, East County Line to STH 164

6) I-43, Walworth County, USH 20 to USH 12

Adjunct Projects -

1) I-94, Waukesha County, West County Line - STH 67

2) Intersection USH 18/STH 164, Waukesha County

3) I-94, Monroe County, Kirby to USH 12

2. Project Descriptions

A description of each of the primary projects in the study, including ADT, original pavement type, maximum aggregate size, and aggregate hardness is given in Table 1, below.

TABLE 1: Primary Project Description

Project	**Base Pavement**	**ADT/ Yr. Const.**	**Max. Size Aggregate (inches)**	**Hardness Region**	**L.A. Wear**
I-43, Waukesha	CRCP	42,200 / 1992	3/8" (9.5 mm)	3	26
I-43, Walworth	JRCP	11,650 / 1993	5/8” (16 mm)	3	27
USH 151, Lafayette	AC over thin-edged PCCP	6350 / 1993	5/8” (16 mm)	2	38
STH 21, Juneau	AC over dense base over PCC	4200 / 1994	3/8" (9.5 mm)	2	31
USH 45, Vilas/Oneida	AC	5940 / 1993	5/8” (16 mm)	1	21
STH 63, Washburn	AC	5872 / 1993	3/8" (9.5 mm)	1	24

3. Test Section Descriptions

The construction of the six primary projects was governed by guidelines contained in the work plan developed for the research study. As noted earlier, the objectives of the study were to evaluate SMA's using the range of stabilizers (both fiber modified and polymer modified) from the standpoint of ease of construction and pavement performance. To ensure that all factors were evaluated, the design configuration shown in Table 2 was specified for each of the six primary projects constructed in the state:

TABLE 2: Project Layout

Test Section	Description
F1	SMA w/ Cellulose Fiber Stabilizer
F2	SMA w/ Mineral Fiber Stabilizer
P1	SMA w/ Polymer (Thermoplastic) Stabilizer (Lo %)
P2	SMA w/ Polymer (Thermoplastic) Stabilizer (Hi %)
E1	SMA w/ Polymer (Elastomeric) Stabilizer (Lo %)
E2	SMA w/ Polymer (Elastomeric) Stabilizer (Hi %)
Control	Dense Graded Asphalt Mix

One of the projects in each "Hardness Region" was to be constructed using a 3/8" (9.5 mm) maximum sized aggregate, while the other project in the same region was to be constructed using a 5/8" (16 mm) maximum sized aggregate. Since each test section was to be a minimum of 4000' (1200 m) in length, a minimum project length of 5.5 miles (8400 m) was needed to construct all sections.

Most other studies previously conducted and reported on SMA pavements up to the time this study started had centered on the documentation of the mix parameters and construction procedures. This study was specifically established to evaluate SMA pavements primarily from the standpoint of ease of construction and in-place performance. Therefore, while briefly discussing mix parameters and construction procedures, this report will tend to concentrate on the performance of the various mixes once in place.

According to the work plan, ease of construction of the different stabilized SMA mixes was to be evaluated using subjective criteria. This was to be accomplished from observations and through discussions with construction project personnel regarding the mixing, hauling, paving and compaction of the SMA mixes.

The work plan also directed that performance of all test and control sections be evaluated from the standpoint of friction, ride, and durability. Durability was to be defined primarily through rutting and cracking measurements, but was also to include other types of pavement deterioration. In addition, an attempt was to be made to determine the effect of the SMA surface on tire noise.

C. CONSTRUCTION OF SMA PROJECTS

1. Mix Design

The design of the SMA mixes differ from the standard mix in several ways. Primarily, the SMA mixes use a gap-graded aggregate, while the standard mixes use an aggregate gradation that is more evenly spread across the gradation band thus producing a more dense mix. The gap-grading of the aggregate in the SMA results in a more open texture, creating stone-on-stone contact with a higher amount of voids in mineral aggregate (VMA) in the mix (See Photos 1-4). The standard AC mix in Wisconsin for these projects averaged 15% VMA, while the SMA mixes had around 17% VMA. Also, the standard AC mix had around 5.3 to 5.6% asphalt binder, while the typical SMA mix had between 6 to 6.7%. The greater amount of asphalt binder in the SMA mixes required that a combination of fines (mineral filler) and a stabilizing agent, usually either a fiber or a polymer, be added to the SMA to keep the asphalt binder from draining from the mixture during the construction process, especially during hauling.

Photograph 1: SMA Surface

Photograph 2: Standard Dense-Graded Surface

Photograph 3: SMA Pavement Core

Photograph 4: Standard Pavement Core

2. Mixture Production and Placement

The initial SMA trial project, constructed in 1991 on I-94 west of Waukesha, Wisconsin, was the first project constructed in the United States and was considered to be an adjunct project for this study. A batch plant was used for preparing all the mixes for this project. All subsequent projects used either a drum or a batch plant, although a drum plant was used most prevalently. All mineral aggregates were fed through conventional bins for both types of plants. Adjustments to the batch plants were minimal, with the fibers and polymers either fed in by hand or on a calibrated conveyor. At the drum plants, the cellulose and mineral fibers were blown in using a special machine calibrated to the speed of the aggregate feed. The polymer pellets were added into the drum mixer through the recycled asphalt pavement (RAP) feed inlet. The opening of the RAP bin was adjusted and calibrated based on time. In the first few projects utilizing drum plants, lime was added through the cold feed belts. However, problems related to the lime being blown out by airwaves in the drum resulted in the lime either being improperly mixed with the asphalt binder or it being removed from the mix entirely and blown into the baghouse. Consequently, adjustments were made to the drum by extending the knock-out plate 18” (46 cm) into the drum to improve the mixing of the lime with the AC. In addition, roughly half the lime for the mix was recycled through the baghouse and fed directly into the drum, along with the collected mineral dust.

Mix temperatures at the plant were kept in the 295° F – 310° F (146° C – 154° C) range, depending on the stabilizer used. The subsequent lay down temperatures were primarily in the 285° F - 300° F (140° C - 149° C) range. Paving was accomplished in 12-foot to 14-foot (3.6 m - 4.3 m) lane widths. Conventional equipment was used in all cases for the entire paving operation. The contractors used either two or three steel rollers following the paver. When two rollers were used, the first roller was 12-ton (10.9 Mg) and the second either a 12-ton (10.9 Mg) or a 10-ton (9.1 Mg) roller. If three rollers were used, the third was placed between the breakdown roller and the cold roller and was generally 12-ton (10.9 Mg). As mentioned previously, in all cases, the rollers used were steel drummed.

The primary difference between paving operations for a standard AC mix and SMA was the length and tightness of the paving train. The openness of the SMA mix results in the mixture losing temperature quite rapidly; therefore, it was important to keep the hot roller close behind the paver to ensure adequate compaction. Ideally, the hot roller was kept within 300' (90 m) of the paver, making three to five passes. The second roller also made three to five passes and was placed 200’ - 300' (60 m - 90 m) behind the hot roller. Thus, the entire paving train was about 500’ - 600' (150 m - 185 m) in total length. If three rollers were used, the length of the train was simply extended another 300' (90 m).

The importance of spacing and location of the rollers was demonstrated early in the construction of the study projects. In one of the first test sections built, conventional rolling methods were used, i.e., the hot roller 700’ (215 m) behind the paver and the cold roller 2500’ (760 m) behind the paver. Field densities in this portion, as measured with a nuclear densiometer, ranged from 87% to 90%. After switching to the aforementioned roller spacings, densities rose to 91% to 93%. (Note: Based upon WisDOT's experience with laboratory mixes at that time, the SMA target field densities were set at 92% and those for the standard mixes were in the 91% to 92% range. Since that time, guideline specifications developed by the Federal Highway SMA Technical Working Group have set the target density for SMA's to 94%, based predominantly on core density data. WisDOT currently tests for densities using a test strip method involving both methods of density determination – nuclear densiometer and cores.) As a matter of fact, when densities were measured just behind the paver with the tamper bars turned on, densities of 89% were achieved. Thus, it seems obvious that most compaction of SMA mixes results from a combination of having the paver tamper bars on and making sure that the initial or breakdown roller is kept tight behind the paver.

3. Construction Problems

Some of the problems experienced during placement have been previously mentioned and will not be reiterated here. Other problems encountered relate to mixture temperature and the sensitivity of SMA's to low ambient temperatures during placement. Proper temperatures were especially critical for the polymer stabilized SMA mixtures. At lay down temperatures much above 305° F (152° C) these mixes tended to bleed, with the polymer pooling and solidifying at the pavement surface (See Photos 5 & 6). On the other hand, at temperatures below 290° F (143° C) the mix would tend to stick in the truck boxes. The elastomeric and thermoplastic stabilized SMA's tended to be much more sensitive than the fiber stabilized mixes regarding these characteristics.

Photograph 5: Bleeding of SMA Pavement

Photograph 6: Bleeding of SMA Pavement

The SMA pavement acted much like the standard asphalt pavement in most other respects. However, either because of the greater amount of asphalt binder or the openness of the SMA mix it seemed to cool and become stiffer more rapidly than the standard mix. This temperature sensitivity of the SMA mix required that loads be continually delivered to the paving operation to ensure that the paving train would be kept constantly moving. Any delay that caused the paver to stop resulted in the development of a bump on the pavement surface that was much more prominent with the SMA mix than for a standard AC mix. This was presumably a result of the mix cooling and the paver screed subsequently riding up over the cooled mix when the paver started moving again.

Handwork was also more difficult with the SMA than with the standard mix. This was attributed to the coarseness of the mix, making it more difficult to handle, and its tendency to cool quickly. Tools had to be constantly cleaned due to the tackiness of the mix. Other equipment, from truck boxes to the plant, needed to be thoroughly cleaned at the end of each day. Also, lay down at ambient air temperatures below 50° F (10° C) greatly exacerbated the problems associated with the temperature sensitivity of the SMA's.

It should be emphasized that the projects included in this study were WisDOT's first SMA pavements. Thus, the contractors had little experience with these mixtures at the time of construction. Over the years, technology and contractor experience have increased, resulting in fewer construction related problems.

D. PAVEMENT PERFORMANCE

1. Performance Parameters

Six main parameters were used to evaluate the effectiveness of each SMA pavement type from the standpoint of overall performance: (1) the amount of cracking; (2) friction; (3) overall pavement distress (PDI); (4) rutting; (5) noise impacts; and (6) ride (using the International Ride Index or IRI).

a) Crack Surveys:

Pre-construction crack surveys were taken on a 0.2 mi (320 m) portion of each test and control section to establish baseline data. Post construction surveys were then taken at regular intervals from the date of construction on the same 0.2 mi (320 m) segments. Any crack longer than 4' (1.2 m) counted as a whole crack. Since the pavements were monitored from the standpoint of overall performance, no attempt was made to distinguish between reflective cracking and fatigue cracking, with all cracks being reported as reflective cracks. In most cases the crack surveys were taken during the coldest months of the year under the assumption that the majority of the cracks would be most easily visible at this time of year. These survey results were then reported in units of *cracks/mile* and in *% of cracks reflected.*

b) Friction Measurements:

Initial friction measurements were taken on all test and control sections three months following construction. Subsequent measurements were taken at regular intervals from the date of construction. Measurements were taken with a ribbed-tire using water sprayed in front of a locked-up wheel. Speed gradients, or the frictional difference between 40 mph (64 km/hr) and 50 mph (80 km/hr) divided by ten, were also calculated.

c) Pavement Distress Index (PDI) Surveys:

PDI surveys were taken on an annual basis on the same segments used for the crack surveys. The same standard PDI procedures and formats used for all pavements in Wisconsin were used on the test and control sections. The PDI surveys were taken during the summer months when the various types of distress for this survey can be safely and accurately acquired. Wisconsin's PDI values range from 0 to 100. Lower numbers indicate a pavement with less distress, and a PDI of around 60 indicates that a pavement is a candidate for maintenance operations.

d) Rut Depth Surveys:

Initial rut measurements were taken as soon after construction as practicable to establish baseline data. Subsequent surveys were taken on an annual basis (at the same time as the PDI surveys). All rut readings were initially recorded to the nearest 0.05" (1.3 mm), but were later measured with a road profiler equipped with electronic rut sensors and recorded to the nearest 0.01" (0.25 mm).

e) Noise Measurements:

Noise readings were taken on two different occasions along the USH 43 project in Waukesha County. Two different measurement techniques were used. The first was the single-vehicle pass-by method with the microphone placed at 50' (15 m) from the roadway, typical of measurements made in the United States. The second technique also used single-vehicle pass-by, but the microphone was placed 25' (7.6 m) from the roadway, as is typical of measurements made in Europe.

f) Ride Measurements:

Ride was measured with a South Dakota Road Profiler and recorded in International Ride Index (IRI) units. These were taken at yearly intervals following the first year after construction.

2. Performance Results

Before summarizing the project results, it should be reemphasized that the SMA pavements included in this study were WisDOT's first generation of SMA's. Over the years, technology has increased, resulting in better specifications and improved SMA mixes. WisDOT is currently using their fourth generation of SMA's; thus, the performance of today's SMA pavements in Wisconsin may be different than the performance of those involved in this study.

In addition, it needs to be made clear that, although this research study began in 1992 with the construction of the first SMA project, the final project built under the auspices of this study was not constructed until 1994. Therefore, data was collected until 1997, 1998 or 1999, depending on the specific project. Also, it should be remembered that the data was collected at various times of the year. Thus, even though the results being reported for all projects represent the experience over the five years dictated by the study plan, the data wasn't necessarily collected in the same calendar year and the cumulative data for the projects in the study are out of sync, with some data representing slightly more or less than the actual five years. Following, then, are the pavement performance results:

a) Crack Data

Tables 3 and 4 present the fifth-year crack information and analysis from the primary projects in this study. The data provided in these tables is expressed in *% of Cracks* using the amount of cracks in the original pavement surface prior to overlaying as the baseline. Table 3 presents a crack analysis of each of the different SMA types after five years performance, and compares the SMA pavements (by type and as a whole) against the standard dense graded asphalt pavement (control). Table 4 presents an analysis comparing the crack data for all six projects after five years performance broken down by the four different study variables, i.e., by SMA Stabilizer, by Maximum Aggregate Size, by Hardness Region and by Base Pavement Type. Figure 2 provides a five-year historical comparison of the overall crack performance for SMA's compared against that of the standard dense-graded asphalt pavements (DGAP) based on crack generation for these two types of pavements over time.

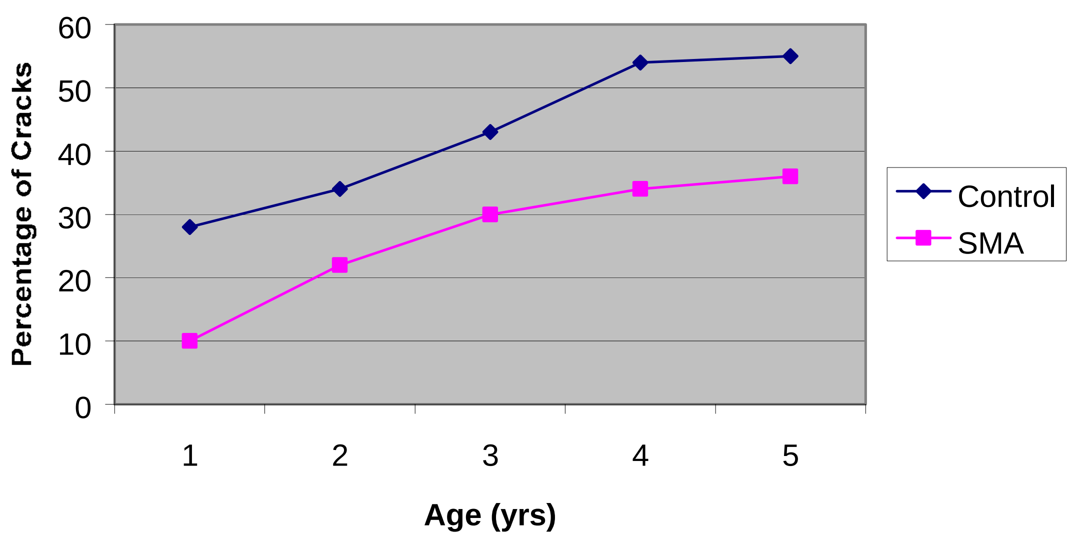
TABLE 3: Crack Analysis at Five Years by Project (% of Cracks)

Test Sections	Control	F1	F2	E1	E2	P1	P2	Mean for SMA's
Projects								
STH 63	69	26	21	15	25	28	38	26
STH 21	78	68	73	86	71	71	65	72
I-43Wauk.	68	60	56	43	39	50	42	48
USH 45	12	6	17	13	14	10	8	11
USH 151	67	56	47	46	48	55	60	52
I-43 Wal.	38	1	6	7	7	8	8	6
Mean	55	36	37	35	34	37	37	36

TABLE 4: Crack Analysis at Five Years by Aggregate Size, Hardness Region and Base Pavement (% of Cracks)

Mixes	3/8" (9.5 mm)	5/8" (16 mm)	Region 1	Region 2	Region 3	Over AC	Over PCC
F1	51	21	16	62	30	33	39
F2	50	23	19	60	31	37	36
E1	47	22	14	66	25	38	32
E2	45	23	20	60	23	37	31
P1	50	24	19	63	29	36	38
P2	48	25	23	62	25	37	37
Mean	49	23	19	62	27	36	36
Control	72	39	40	72	53	53	58

FIGURE 2: Crack Analysis



Results from Table 3 indicate that after five years, from an overall standpoint, the SMA's tested in this study are performing better than the standard mix, i.e., 36% reflective cracking vs 55% for DGAP. Furthermore, Figure 2 strongly demonstrates that this relationship has been fairly continuous over the length of the study period. Results in Table 3 would also suggest that the different types of SMA's are performing pretty much equally, with those constructed using a high percentage of elastomeric polymers (E2) for stabilizers being slightly better, but this difference is extremely marginal.

The results reported in Table 4 suggest that aggregate size appears to have some impact on cracking when comparing the performance of the SMA's with the different sized aggregates against each other. However, this relationship also holds true for the control mix. In all instances, the SMA with the larger aggregate cracked less than the SMA of the same type with the smaller aggregate (ranging from 48% to 59% less) and the control sections in those same projects also cracked less (46% less). On average, the SMA's with the larger sized aggregate, 5/8" (16 mm), had 41% less cracks than their respective DGAP controls, while the SMA's with the smaller sized aggregate, 3/8" (9.5 mm), had only 32% fewer cracks than their respective DGAP controls.

For the 3/8” (9.5 mm) aggregate, the best performing SMA was the one modified with a high percentage elastomeric polymer (E2), while the best SMA for the 5/8” (16 mm) aggregate was the one with the organic fiber (F1) modifier.

Table 4 also demonstrates that the base pavement appears to have marginal, if any, impact. SMA’s placed over AC pavements experienced 32% less cracking compared to the DGAP control and SMA’s placed over PCC pavements experienced 38% less cracking. In this comparison the different types of SMA’s performed pretty much equally. However, the F1 modified SMA’s seemed to have a marginally slight edge over the others when placed over an AC pavement; and, the E1 and E2 modified SMA’s had a slight edge over the others when placed over a PCC pavement.

Not surprisingly, the factor that seemed to affect performance the most was aggregate hardness. In all instances the projects in the region with the hardest aggregate out performed those in regions with softer aggregates and projects in the region with the softest aggregate performed the worst. This holds true for the controls, also. On average, the SMA’s in Region 1 cracked about 53% less than their respective DGAP controls. Those in Region 3 cracked 49% less, while those in Region 2 cracked only 14% less than their respective controls.

Based on the crack results of this study, then, the best performing SMA is one modified with an elastomeric polymer, with 5/8” (16 mm) maximum sized hard aggregate.

b) Friction Data

As was mentioned previously, friction tests were conducted at both 40 mph and 50 mph (64 km/hr and 80 km/hr) using a ribbed tire in the locked position. These tests result in a friction number (FN), which represents the force exerted on the locked wheel as it is dragged along the pavement surface at the particular speed. This is done under simulated wet conditions by spraying water in front of the locked wheel. In accordance with Chapter 14 of WisDOT’s Facilities Development Manual, “…minimum friction numbers of 30 or more are considered desirable for roadways carrying low to moderate traffic volumes while minimum friction numbers of 35 and above are appropriate for those carrying higher traffic volumes." From the FN's at 40 mph and 50 mph (64 km/hr and 80 km/hr) the speed gradients were also computed. The speed gradient is the difference in frictional values at those two speeds divided by the difference of those two speeds, which is ten in this case. The numbers are unitless and any number above a 0.4 is not desirable as it represents a significant difference in frictional characteristics at those two speeds. On the other hand, speed gradients less than 0.2 represent a much safer condition where frictional characteristics for braking are pretty much uniform over the speed range. Tables 5, 5a, 6 and 6a provide the friction analysis of the six primary projects after five years. Tables 5 and 6 present the analysis in terms of FN at 40 mph (64 km/h), while Tables 5a and 6a present the friction characteristics in terms of the speed gradient.

TABLE 5: Friction Analysis at Five Years [FN @ 40 mph (64.4kph)]

	Region 1		Region 2		Region 3		
Agg. Size	3/8" (9.5 mm)	5/8" (16 mm)	3/8" (9.5 mm)	5/8" (16 mm)	3/8" (9.5 mm)	5/8" (16 mm)	
Section	STH 63	USH 45	STH 21	USH 151	I-43 Wauk	I-43 Walw	Mean
F1	50	51	45	40	39	41	44
F2	46	50	49	42	40	43	45
E1	47	50	47	44	38	44	45
E2	50	48	47	42	39	44	45
P1	48	52	44	44	40	44	45
P2	48	51	48	45	38	44	46
Mean	48	50	47	43	39	43	45
Control	44	54	49	40	48	52	48

TABLE 5a: Friction Analysis at Five Years (Speed Gradient)

	Region 1		Region 2		Region 3		
Agg. Size	3/8" (9.5 mm)	5/8" (16 mm)	3/8" (9.5 mm)	5/8" (16 mm)	3/8" (9.5 mm)	5/8" (16 mm)	
Section	STH 63	USH 45	STH 21	USH 151	I-43 Wauk	I-43 Walw	Mean
F1	0.25	0.16	0.14	0.20	0.30	0.19	0.21
F2	0.26	0.14	0.39	0.22	0.31	0.22	0.26
E1	0.25	0.18	0.20	0.20	0.15	0.31	0.22
E2	0.26	0.12	0.25	0.24	0.15	0.35	0.23
P1	0.24	0.12	0.19	0.18	0.32	0.21	0.21
P2	0.22	0.13	0.35	0.20	0.09	0.18	0.20
Mean	0.25	0.14	0.25	0.21	0.22	0.24	0.22
Control	0.32	0.25	0.33	0.21	0.36	0.29	0.29

TABLE 6: Friction Number at Five Years by Aggregate Size and Hardness Region (From data in Table 5)

Mixes	3/8" (9.5 mm)	5/8" (16 mm)	Region 1	Region 2	Region 3
F1	45	44	50	42	40
F2	45	45	48	46	42
E1	44	46	48	46	41
E2	45	45	49	44	42
P1	44	47	50	44	42
P2	45	48	50	46	41
Mean	45	46	49	45	41
Control	47	49	49	45	50

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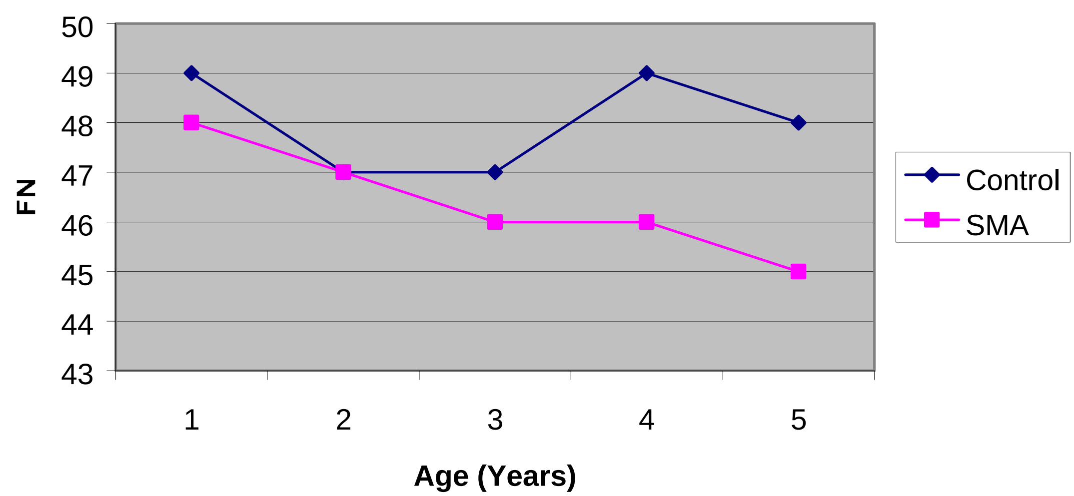
TABLE 6a: Speed Gradient at Five Years by Aggregate Size and Hardness Region (From data in Table 5a)

Mixes	3/8" (9.5 mm)	5/8" (16 mm)	Region 1	Region 2	Region 3
F1	0.23	0.19	0.20	0.17	0.24
F2	0.32	0.19	0.20	0.30	0.26
E1	0.20	0.23	0.22	0.20	0.23
E2	0.22	0.24	0.19	0.24	0.25
P1	0.25	0.14	0.18	0.18	0.26
P2	0.22	0.17	0.18	0.28	0.14
Mean	0.24	0.19	0.20	0.23	0.23
Control	0.34	0.25	0.29	0.27	0.33

FIGURE 3: Friction Analysis



The FN data indicates that the SMA's have slightly lower overall FN values than the standard mix after five years performance, 45 versus 48, respectively. Indeed, as can be seen in Figure 3, this relationship has existed from the beginning and the gap between the two types of pavements appears to be increasing over time. On the other hand, overall speed gradient numbers seem to favor the SMA's, 0.22 for the SMA's and 0.29 for the DGAP's. Also, while aggregate size doesn’t seem to have much impact on FN, hardness, as would be expected, does with the harder aggregates providing the higher FN values. The exception to this observation occurs in the projects of Region 3. The FN values for the SMA sections in these projects average 41, noticeably less than those in the region with the softer aggregate (Region 2), but still considered good. However, the FN values for the control sections in these same projects in Region 3 remain quite high at an average of 50. The only explanation offered is that the projects in Region 3 have from two to ten times more traffic on them than the projects in the other regions resulting in accelerated wear, and the aggregate in the DGAP mixes might not wear as fast because more aggregate is in contact with the tire surface in these sections.

The data results from the standpoint of speed gradient would indicate that aggregate size and hardness does affect performance. From the results in Tables 5a and 6a, it can be seen that the harder aggregate and larger aggregate provide the best safety characteristics relative to breaking. As can be seen from the data in these tables, the SMA with the best frictional characteristics seems to vary from project-to-project. However, if one were to be picked as the best performer from the standpoint of friction, an SMA with a 5/8” (16 mm) maximum, harder aggregate and modified with a high percentage of thermoplastic polymer (P2) may have a slight edge over the others, but it is really too close to call with any certainty.

c) Pavement Distress Data

As discussed earlier, the PDI rating is an indication of the overall distress of a pavement. WisDOT uses a PDI range of 0 to 100. This is a unitless number where lower numbers represent good pavements and higher numbers represent pavements that have a greater amount of distress. As mentioned previously, a PDI of 60 or higher indicates that the pavement should be considered for some kind of remediation. Tables 7 and 7a provide PDI data for the projects in the fifth year of pavement age.

TABLE 7: Pavement Distress Index Analysis at Five Years

	Region 1		Region 2		Region 3		
Aggregate	3/8" (9.5 mm)	5/8" (16 mm)	3/8" (9.5 mm)	5/8" (16 mm)	3/8" (9.5 mm)	5/8" (16 mm)	
Section	STH 63	USH 45	STH 21	USH 151	I-43 Wauk	I-43 Walw	Mean
F1	32	7	27	36	32	10	24
F2	47	32	27	19	33	18	29
E1	13	34	6	19	14	27	19
E2	13	13	27	20	6	15	16
P1	13	13	27	26	14	6	17
P2	23	17	6	32	27	31	23
Mean	24	19	20	25	21	18	21
Control	48	13	27	30	38	47	34

TABLE 7a: PDI Analysis at Five Years by Aggregate Size and Hardness Region (From data in Table 7)

Mixes	3/8" (9.5 mm)	5/8" (16 mm)	Region 1	Region 2	Region 3
F1	30	18	20	32	21
F2	36	23	40	23	26
E1	11	27	24	12	20
E2	16	16	13	24	10
P1	18	15	13	26	10
P2	19	27	20	19	29
Mean	22	21	22	23	19
Control	38	30	30	28	42

From this data it can be seen that most of the test sections are still in relatively good condition. The SMA's (with an average PDI of 21) are performing 38% better than the control pavements (averaging a PDI of 34). Neither aggregate hardness nor aggregate size appears to have much impact on overall performance. Furthermore, it appears that the stabilizer used in the SMA's doesn't have much impact on overall performance either, even though the SMA with the elastomeric polymers (E1 and E2) and the low percentage thermoplastic (P1) stabilizers may be performing slightly better than the others. The results of this part of the study, coupled with the results for the Crack analysis, would indicate a trend that suggests SMA pavements may have a slightly longer life than a standard DGAP.

d) Rut Depth Data

Rut depth values were measured with a South Dakota type road profiler using sensors located in each wheel path and in the center of each lane. The data given in Table 8 represents five year's experience. As can be seen in the table, the data is inconclusive because of the uniformly low values for all sections, including that for the control sections.

Table 8: Rut Depth Analysis at Five Years (units in inches)

	Region 1		Region 2		Region 3		
Agg. Size	3/8" (9.5 mm)	5/8" (16 mm)	3/8" (9.5 mm)	5/8" (16 mm)	3/8" (9.5 mm)	5/8" (16 mm)	
Section	STH 63	USH 45	STH 21	USH 151	I-43 Wauk	I-43 Walw	Mean
F1	0.08	0.12	0.12	0.12	0.12	0.08	0.12
F2	0.08	0.20	0.12	0.12	0.12	0.08	0.12
E1	0.08	0.16	0.12	0.12	0.16	0.08	0.12
E2	0.08	0.16	0.12	0.12	0.08	0.12	0.12
P1	0.12	0.16	0.08	0.12	0.08	0.12	0.12
P2	0.08	0.16	0.08	0.12	0.12	0.08	0.12
Mean	0.08	0.16	0.12	0.12	0.12	0.08	0.12
Control	0.08	0.16	0.12	0.08	0.08	0.12	0.12

e) Ride Data

Ride data was also taken with the road profiler described above and the results are shown in Table 9. These readings were recorded in International Ride Index (IRI) values, in the standard units of meters/kilometer. Values in the 3.0 to 3.5 range or greater are considered to be rough, while those less than 2.0 are considered to be relatively smooth. Although none of the pavements are considered rough, in general the SMA's seem to be rougher than the control standard mix; and, as would be expected, the SMA's with the larger sized aggregate are not quite as "smooth" as the SMA's with the smaller sized aggregate.

Table 9: Ride Analysis at Five Years (IRI) (units in m/km)

	Region 1		Region 2		Region 3		
Agg. Size	3/8" (9.5 mm)	5/8" (16 mm)	3/8" (9.5 mm)	5/8" (16 mm)	3/8" (9.5 mm)	5/8" (16 mm)	
Section	STH 63	USH 45	STH 21	USH 151	I-43 Wauk	I-43 Walw	Mean
F1	1.15	1.68	1.23	1.49	1.24	1.07	1.31
F2	1.16	2.52	1.12	1.58	1.38	1.36	1.52
E1	1.43	2.49	1.16	1.47	1.17	1.35	1.51
E2	1.50	2.66	1.54	1.78	1.21	1.15	1.64
P1	1.19	3.10	1.37	1.66	1.04	1.32	1.61
P2	1.30	2.72	1.19	1.50	1.24	1.14	1.52
Mean	1.29	2.53	1.27	1.58	1.21	1.23	1.52
Control	0.93	1.18	1.26	1.25	1.17	0.71	1.08

f) Noise Data

The noise data collected from the test and control pavements was analyzed across the spectrum within the range of human hearing. The averaged or weighted noise level for both the standard asphalt pavement and the SMA was the same. However, in the area most sensitive to the human ear, roughly between 1500-2000 Hz, the trace for the SMA dropped below that of the standard asphalt pavement. This drop amounted to about 5 dBA and gave the perception that the SMA was slightly quieter.

g) Other Factors

Other factors have been noted that offer additional and important information. First there is the impact of the open texture of the SMA's on the application of traffic paint. Traffic paint applied to SMA's appeared to have a mottled look. However, applying roughly 50% more thickness of paint seemed to correct the problem. The use of tape rather than paint had mixed reviews. The tape worked well on some SMA's, but wouldn't stick to other SMA's, perhaps due to the low surface area.

Next, there is the impact on salt usage. It was found that while the SMA surfaces initially required more salt, the openness of the texture seemed to hold it longer. Whether or not this results in a net gain or loss in salt usage wasn't able to be determined due to the shortness of the test sections.

In addition, observations of vehicle spray indicate that the SMA surfaces do not spray or splash water as much as the standard AC pavement does during a rain event. However, again because of the open texture of the SMA, water is held in the pores longer after an event, resulting in the presence of a slight spray in the test sections long after the control section was dry. Related to this are observations made by the traveling public. Motorists traveling these test sections have reported that the standard dense graded pavement tends to "glisten like a mirror in the rain at night", while the SMA sections do not. Highway engineers view the "glistening" as a significant safety concern, because it tends to adversely affect motorists traveling in inclement weather, when visibility is already hampered. Under these conditions, the SMA pavements tend to be favorable because they reflect less light.

Finally, it should be kept in mind that SMA's generally cost from 10% to 15% more than the standard AC pavement. This increased cost is primarily due to the increased amount of asphalt binder used in these mixes and the necessity for adding stabilizers to help hold that increased AC in the mix. This does not account for any possible increases or decreases in maintenance costs that may result.

E. CONCLUSIONS

Before proceeding with the conclusion, it should be recognized that there were problems associated with the placement of the SMA test pavements that may have affected their performance. These included a range of problems such as contractors not being accustomed to maintaining a tight paving train, as has been noted earlier in this report, and difficulty in maintaining a proper mix temperature at the paver because of either a long haul from the plant or low ambient air temperatures. There were other problems that always seem to occur when constructing a research project of this nature and by the time the problem is solved, the next test section is ready to be constructed. Nevertheless, it is felt that if SMA pavements are to be accepted as a reliable and effective surface they should be somewhat forgiving and be able to perform adequately even though they may have been placed under less than ideal conditions. Therefore, the data collected from all of the test pavements was used regardless of the construction problems experienced. The only data not used was where it may have been erroneous due to a failure or problem with the data collection equipment. With that said, from the data presented in this report, the following conclusions can be drawn:

- The SMA’s are providing improved service from the standpoint of cracking, which has been 30% to 40% less for the SMA’s than for the standard DGAP’s in most instances.
- The larger maximum size aggregate SMA’s seem to have impeded crack development more than the smaller sized aggregate SMA’s.
- The SMA’s in the region with the aggregates most resistant to abrasion and impact retard cracks 52% better than the standard AC pavements, while those in the region with aggregates least resistant to abrasion and impact retard cracks only 14% better than the standard AC control.
- SMA’s placed over another AC pavement seem to crack at the same rate as do those placed over PCC pavements. In both instances, the SMA’s seem to be providing roughly 35% better crack resistance than the standard AC control.
- After five years, the SMA pavements are providing less friction but better speed gradients than the standard asphalt pavement. No one SMA type is better than another in this regard. This fact, combined with the subjective observations of spray and glare, suggest that SMA pavements may have only slightly better safety benefits than the standard asphalt pavements.
- Considering all types of distress, the SMA’s seem to be better performers than the standard AC pavements over the five-year study period. In this regard, the different types of SMA’s seem to be performing almost the same, although the SMA modified with the inorganic fiber (F2) seems to be the worst performing SMA.

- At this stage, the SMA pavements do not seem to be providing any significantly better rut resistant capability than the standard asphalt mix used in Wisconsin. Both types of pavements are very good performers.
- The analysis of ride values indicates that an SMA is generally rougher when compared with a standard AC pavement. Since the mean of the readings for the SMA's with the 5/8" (16 mm) sized aggregate are worse than those of the 3/8" (9.5 mm) sized aggregate, it can be presumed that this may be due to the electronic equipment "reading" the open texture as being "rougher".

While no hard evidence is forthcoming from this project at this time, the trend suggests that a SMA pavement may have a slightly longer life. While SMA's evaluated in this study are performing better than the standard AC pavements in the important areas of crack and distress generation, their overall cost-effectiveness compared to a standard dense-graded asphalt pavement is unknown at this time.

It should be reiterated that the SMA pavements evaluated in this study were WisDOT's first generation of SMA's. Thus, WisDOT and the contractors had little experience with these mixtures at the time of construction. Over the years, technology has improved, locally and nationally, resulting in better specifications, improved SMA mixes and fewer problems associated with construction. WisDOT is currently using their fourth generation of SMA's; thus, the performance of today's SMA pavements in Wisconsin may be different than the performance of those involved in this study.